RESPONSE OF TALL STEEL BUILDINGS IN SOUTHERN CALIFORNIA TO THE MAGNITUDE 7.8 SHAKEOUT SCENARIO EARTHQUAKE

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Abstract

Currently, there is a significant campaign being undertaken in southern California to increase public awareness and readiness for the next large earthquake along the San Andreas Fault, culminating in a large-scale earthquake response exercise. The USGS ShakeOut scenario is a key element to understanding the likely effects of such an event. A source model for a M7.8 scenario earthquake has been created (Hudnet *et al.* 2007), and used in conjunction with a velocity model for southern California to generate simulated ground motions for the event throughout the region (Graves *et al.* 2008). We were charged by the USGS to provide one plausible realization of the effects of the scenario event on tall steel moment-frame buildings. We have used the simulated ground motions with three-dimensional non-linear finite element models of three buildings (in two orthogonal orientations and two different connection fragility conditions, for a total of twelve cases) in the 20-story class to simulate structural responses at 784 analysis sites spaced at approximately 4 km throughout the San Fernando Valley, the San Gabriel Valley and the Los Angeles Basin. Based on the simulation results and available information on the number and distribution of steel buildings, we have recommended that the ShakeOut drill be planned with a damage scenario comprising of 5% of the estimated 150 steel moment frame structures in the 10-30 story range collapsing (8 collapses), 10% of the structures red-tagged (16 red-tagged buildings), 15% of the structures with damage serious enough to cause loss of life (24 buildings with fatalities), and 20% of the structures with visible damage requiring building closure (32 buildings with visible damage and possible injuries). This paper details the analytical study underlying these recommendations.

Introduction

In order to prepare for the next big earthquake on the San Andreas fault, the US Geological Survey (USGS) has started a year-long "DARE TO PREPARE" campaign that will culminate in the Great Southern California Shakeout Scenario in 2008, a large-scale earthquake response exercise. A magnitude 7.8 earthquake on the southern San Andreas Fault has been chosen as the scenario event and a detailed, realistic source model for the event has been generated (Hudnet *et al.* 2007) and used to create simulated ground motions at locations throughout Southern California (Graves *et al.* 2008). In support of this effort, we were charged by the USGS with developing a plausible realization of the response of tall steel buildings to the scenario event. With this in mind, we analyze three steel moment frame buildings in the 20-story class, orienting them in two different directions, considering perfect and imperfect realizations of beam-to-column connection behavior, subjecting them to the simulated 3-component ground motions at each of 784 analysis sites in the San Fernando Valley, the San Gabriel Valley and the Los Angeles Basin spaced at approximately 4 km, as shown in Figure 1. We use the modeled building performance in these 12 cases (3

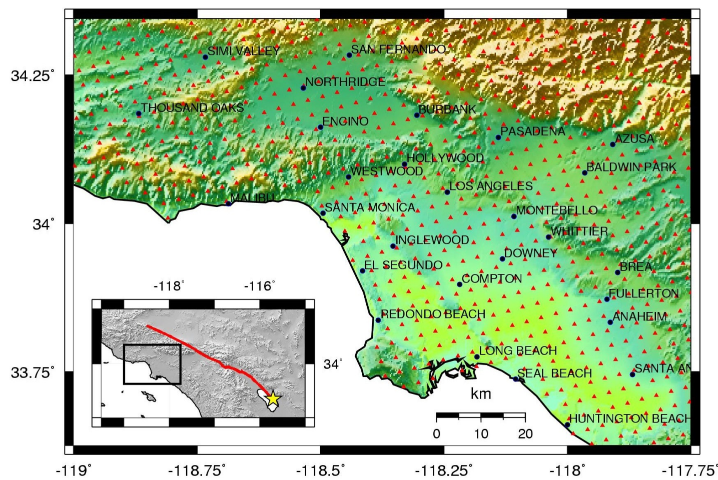


Figure 1: Geographical scope of study area. Triangles represent sites were building time-history analyses are performed. The inset shows the study area in relation to the rupture trace. The star represents the epicenter of the earthquake.

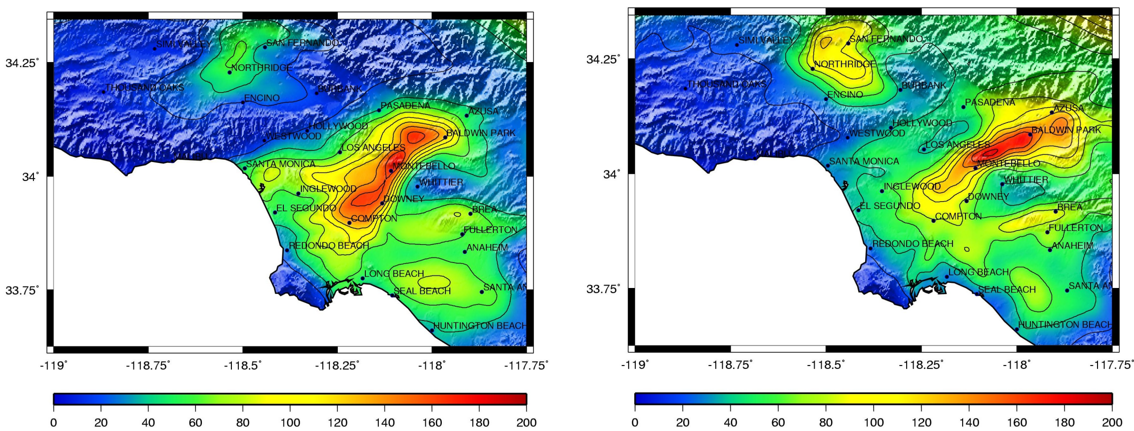
buildings x 2 orientations x 2 connection susceptibility realizations) to provide a qualitative picture of one plausible outcome in the event of the big one striking southern California.

Scenario Earthquake

The scenario earthquake chosen is a magnitude 7.8 earthquake on the San Andreas fault with rupture initiating at Bombay Beach and propagating northwest a distance of roughly 304 km, terminating at Lake Hughes near Palmdale, as shown in the inset of Figure 1. The source model developed by Hudnet *et al.* (2007) is based on a wide variety of observations and constraints. Using this source model, Graves *et al.* (2008) has simulated 3-component seismic waveforms on a uniform grid covering southern California. The SCEC Community Velocity Model (Magistrale *et al.* 1996; Magistrale *et al.* 2000; Kohler *et al.* 2003), which allows for the modeling of the basin response down to a shortest period of approximately 2 s, was used for the ground motion simulations. Figures 2(a) and 2(b) show the peak velocities of the simulated waveforms in the east-west and north-south directions, respectively. Peak velocities are in the range of 0-100 cm/s in the San Fernando valley, and 60-180 cm/s in the Los Angeles basin. Corresponding peak displacement ranges are 0-100 cm and 50-150 cm.

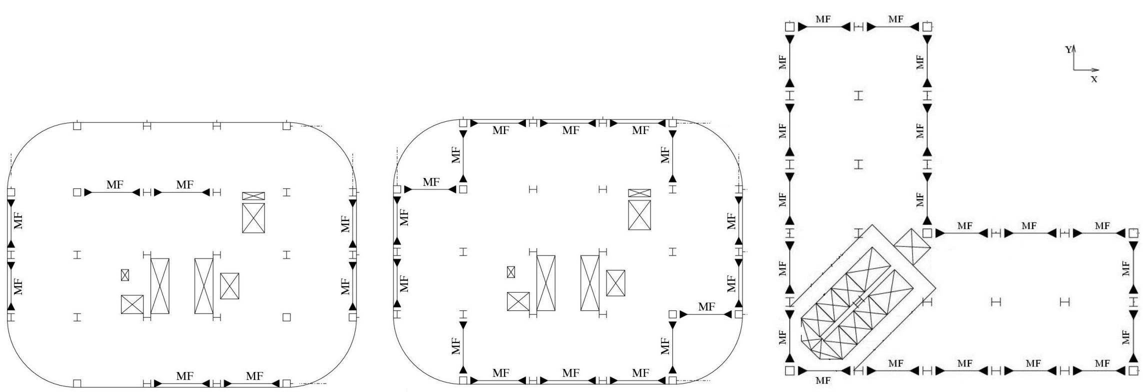
Description of Modeled Buildings

Structural models of three buildings are subjected to ground motions at the 784 analysis sites. Building 1 is based on an existing 18-story office building located within five miles of the epicenter of the 1994 Northridge earthquake. It was designed according to the lateral force requirements of the 1982 UBC and construction was completed in 1986-87. The lateral force-resisting system consists of two-bay welded steel moment-frames, as shown in Figure 3(a). The location of the north frame one bay inside of the perimeter gives rise to some torsional eccentricity. Many moment-frame beam-column connections in the building fractured during the Northridge earthquake, and the building has been extensively investigated since then by many engineering research groups (SAC 1995). Building 2 is similar to building 1, but the lateral force-resisting system has been redesigned according to the 1997 UBC. The new building has been designed for larger earthquake forces and greater redundancy in the lateral force-resisting system and the torsional eccentricity seen in Building 1 has been eliminated. Building 2 has 8 bays of moment-frames in each direction, as shown in Figure 3(b). It is L-shaped in plan, as shown in Figure 3(c). The UBC classifies such buildings as irregular and stipulates that they be designed for lateral forces that are approximately 10% larger than those prescribed for regular buildings. Detailed floor plans, beam and column sizes, and the gravity, wind and seismic loading criteria are given in Krishnan *et al.* 2006 for Buildings 1 and 2 and in Krishnan (2003a, 2007) for Building 3.



(a) (b)

Figure 2: Peak ground velocities (in cm/s) for simulated ground motions in the (a) east-west and (b) north-south directions.



(a) (b) (c)

Figure 3: Typical floor plans are shown for the three buildings modeled: (a) Building 1, an existing 18-story building designed to the 1982 UBC; (b) Building 2, a re-designed version Building 1 conforming to the 1997 UBC; and (c) Building 3, a 19-story L-shaped building designed according to the 1997 UBC. Bays marked “MF" indicate moment frames.

Finite Element Analysis

Nonlinear damage analyses of the structures are performed using the program FRAME3D (Krishnan 2003b). FRAME3D (http://www.frame3d.caltech.edu) incorporates geometric nonlinearity, which enables the modeling of the global stability of the building, accounting for $P - \Delta$ effects accurately. Beams are modeled using segmented elastofiber elements, with nonlinear end segments that are subdivided in the cross-section into a number of fibers, and an interior elastic segment, Column elements include and additional nonlinear segment in the middle to enable modeling of column buckling. Beam-to-column joints are modeled in three dimensions using panel zone elements. These elements been shown to simulate damage accurately and efficiently (Krishnan and Hall 2006a, 2006b). Material nonlinearity resulting in flexural yielding, strain-hardening, and ultimately rupturing of steel at the ends of beams and columns, and shear yielding in the joints (panel-zones) is included. The fracture mode of failure is also included, and used here to consider the effect fracture-susceptible connections on overall building response. There is great uncertainty in the performance of the beam-to-column connections in welded steel moment frame buildings as evidenced in the 1994 Northridge earthquake. Two models are considered for each building, one with perfect connections, and the other with susceptible connections. Specifications (FEMA 2000a) developed by the Federal Emergency Management Agency (FEMA) for moment-frame construction following the Northridge earthquake should result in superior connection performance, and hence, the connections in the buildings designed according to UBC97 are assumed to be less vulnerable to fracture than for the older (pre-1994) Building 1.

Building Performance

At each site, analyses were performed using FRAME3D for the three building models, with perfect and fracture-susceptible connections and in two different orientations (with the x-axis in Figure 3 oriented in the east-west direction and rotated 90 degrees for Buildings 1 and 2 and 45 degrees for Building 3) for a total of 9408 3-D nonlinear time-history analyses. In each case, detailed structural damage as well as the displacements and interstory drifts. To assess the performance of these buildings, we use the performance levels defined by FEMA 356 (FEMA 2000b): Immediate Occupancy (IO), where very limited structural damage has occurred; Life Safety (LS), a damage state that includes damage to structural components but retains a finite margin against collapse; and Collapse Prevention (CP), a damage state at which the structure continues to support gravity loads but retains no margin against collapse. For existing buildings, the interstory drift limits for the IO, LS, and CP performance levels specified by FEMA are 0.007, 0.025, and 0.05, respectively. In addition to these criteria, we assume that the buildings will be red-tagged (RT) if the peak interstory drift ratios exceed 0.05. If the peak interstory drift ratio exceeds 0.075 we assume that there is a great likelihood that the building has collapsed (CO).

Maps of peak interstory drift ratios for the base orientation for the three buildings assuming fracture-susceptible connections are shown in Figures 4(a), 4(c) and 4(e). Corresponding maps assuming perfect connections are shown in Figures 4(b), 4(d) and 4(f). The color-coding on the maps follows the previously-described performance criteria.

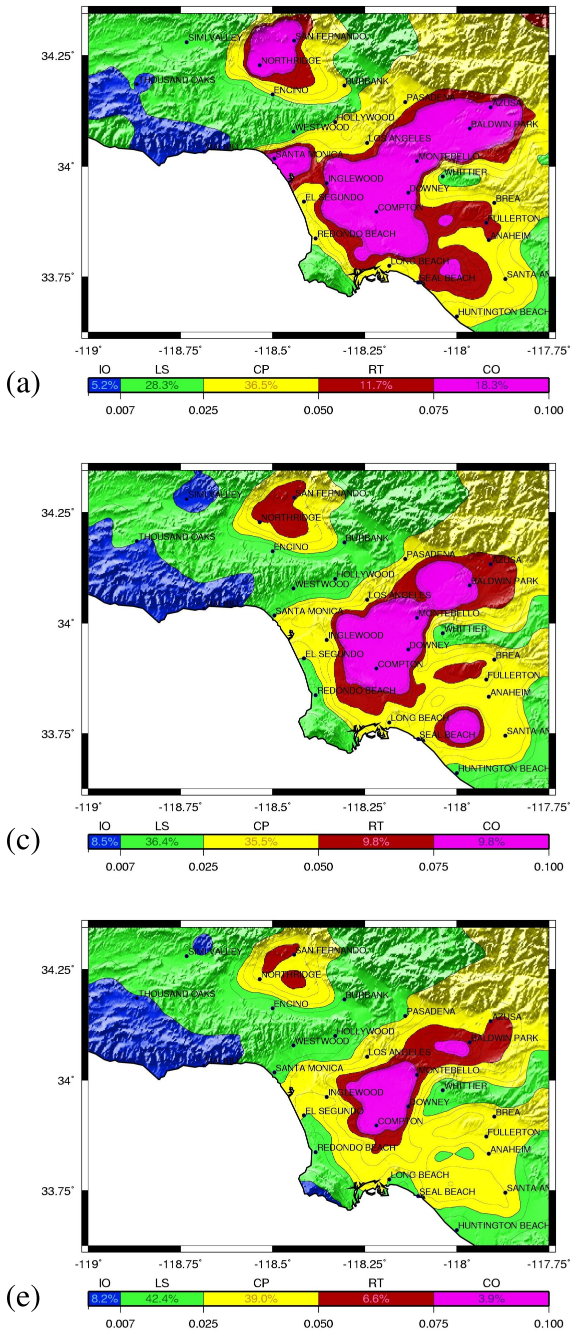


Figure 4: Maps of peak interstory drift for Building 1 with (a) susceptible and (b) perfect connections, Building 2 with (c) susceptible and (d) perfect connections, and Building 3 with (e) susceptible and (f) perfect connections. Color-coding corresponds to performance classification: Immediate Occupancy (IO); Life-Safety (LS); Collapse Prevention (CP); Red-Tagged (RT); Collapse (CO).

Results for building performance are summarized in Table 1. Building 1 exhibits the worst performance with the susceptible connection model collapsing at 18.3% of the analysis sites and being red-tagged at 11.7% of the sites. The L-shaped building 3 performs the best

Model	Orientation	Connections	Performance Level				
			IO	LS	CP	RT	CO
Building 1 (1982 UBC)	Base	Susceptible	5.2	28.3	36.5	11.7	18.3
		Perfect	5.4	29.7	46.0	11.9	7.0
	Rotated	Susceptible	4.8	29.7	33.8	7.5	24.2
		Perfect	4.9	31.0	42.2	10.7	11.3
Building 2 (1997 UBC)	Base	Susceptible	8.5	36.4	35.5	9.8	9.8
		Perfect	8.5	37.2	42.0	7.7	4.7
	Rotated	Susceptible	7.7	36.0	36.0	8.2	12.1
		Perfect	7.7	37.4	41.2	10.0	3.8
Building 3 (1997 UBC) (L-Shaped)	Base	Susceptible	8.2	42.4	39.0	6.6	3.9
		Perfect	8.2	42.8	40.9	6.6	1.5
	Rotated	Susceptible	9.9	45.5	34.2	4.6	5.7
		Perfect	9.9	46.0	35.9	5.5	2.7

Table 1: Building performance in base and rotated orientations, with susceptible and perfect beam-to-column connections. Numbers indicate the percentage of analysis sites at which performance can be categorized as: (a) immediately occupiable (IO); (b) life-safe (LS); (c) collapse-prevention (CP); (d) red-tagged (RT); or (e) collapsed (CO).

with the percentage of collapsed and red-tagged instances being 10.3% and 6.4%, respectively. The performance of building 2 is only slightly worse than building 3. If we assume that the beam-to-column connections are perfect, then there is a significant drop in the number of collapsed and red-tagged buildings. In the rotated orientation, performance is slightly worse for buildings 1 and 2 and slightly better for Building 3, as shown in Table 1. However, the spatial contours of building performance in the corresponding peak interstory drift maps are not significantly altered from those shown in Figures 4(a)-4(f).

Conclusions

The location of tall buildings in the Los Angeles metropolitan area with 10 or more stories is shown in Figure 5. The size of circles shown in the figure is proportional to the number of stories. There are 489 buildings with 10-19 stories, 118 buildings with 20-29 stories, 28 buildings with 30-39 stories, 11 buildings with 40-49 stories, and 10 buildings with 50 or more stories. Many more are in the planning stages or under construction. Its clear that majority (607) are in the 10-30 story range. While a wide variety of structural systems are used in the buildings shown, we assume that approximately one-quarter of these buildings (150) utilize steel moment frames as the primary lateral force resisting system, similar to the buildings considered in this study. The buildings are clustered in small pockets that are aligned with the major freeways in the region. Most tall buildings have been built along Interstate freeway I-10 from Santa Monica to downtown Los Angeles, in the mid-Wilshire district along Wilshire boulevard, and along State Highway 101 from Hollywood to Canoga Park in the San Fernando valley. In addition a few tall buildings are located along Interstate freeways, I-5 and I-405.

Figures 4(a) and 4(b) indicate that performance of the oldest design, Building 1, along the I-10, the Santa Ana-Anaheim corrider and Long Beach generally is classified as CP, with damage serious enough to cause loss of life, but without complete collapse. For Bulidings 2 and 3, designed with UBC97, performance along much of the I-10 is improved to the LS damage state, though downtown Los Angeles remains classified as CP, as shown in Figures 4(c)-4(f). It is important to note that areas in the CP zone are within 10 km of the RT and CO zones. What this means is that given a different set of earthquake source parameters, it is entirely possible that at least some of these locations may end up in the red or pink zones indicating collapses or the need for red-tagging. As shown in Krishnan *et al.* (2006) differences in the hypocenter location, slip distribution, rupture directivity, and the velocity model result in a dramatically different distribution of building damage. Bearing this in mind, we have recommended that the ShakeOut drill be planned with a damage scenario comprising of 5% of the estimated 150 steel moment frame structures in the 10-30 story range collapsing (8 collapses), 10% of the structures red-tagged (16 red-tagged buildings), 15% of the structures with damage serious enough to cause loss of life (24 buildings in the yellow zone with fatalities), and 20% of the structures with visible damage requiring building closure (32 buildings with visible damage and possible injuries).

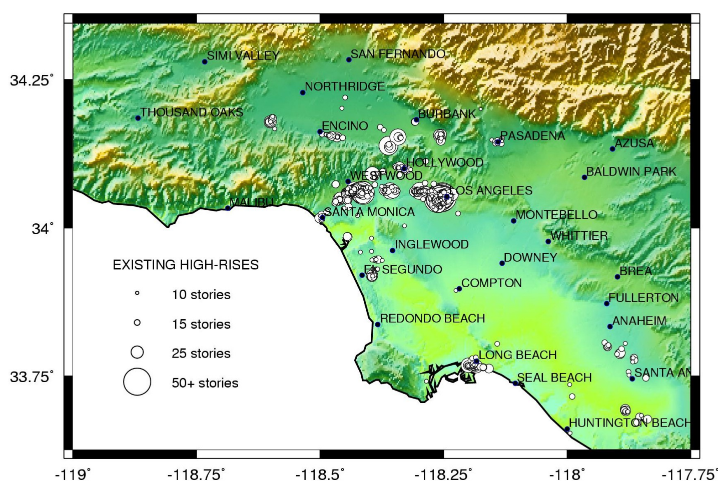


Figure 5: Distribution of tall buildings (10 stories or greater) in the Los Angeles metropolitan area as of mid-2007. Data source: Emporis.com by way of Keith Porter, University of Colorado at Boulder.

Acknowledgements

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